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Seismic design procedures for concentrically braced frames

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This paper deals with the main behavioural issues involved in the seismic design of typical forms of concentrically braced frames. The response of bracing members under monotonic and cyclic axial loading is first considered, and the key parameters affecting the performance are highlighted. This is followed by an assessment of the different design approaches employed for the treatment of brace buckling, and the influence of the underlying assumptions on various aspects of frame response. The provisions of international codes of practice are examined within the discussions, with emphasis on the recommendations of Eurocode 8. Selected results from static and dynamic non-linear analysis on idealised frame configurations are presented in order to illustrate salient response criteria, particularly those related to ductility demand and inelastic distribution. The primary sources of inconsistency existing in code provisions are demonstrated, and their implications on the seismic performance of concentrically braced frames are addressed. It is shown that several code modifications are required in order to alleviate causes of undesirable performance and to facilitate a rational implementation of capacity design concepts.

1. INTRODUCTION

Concentrically braced frames represent a most economical structural form for providing lateral seismic resistance. Because of their geometry, they provide complete truss action, with members subjected primarily to axial forces. Owing to the relatively high stiffness supplied by the braces, this type of frame is very effective in limiting lateral drifts. Consequently, in many applications, braced frames may be favoured over moment-resisting configurations, which are susceptible to large lateral deformations during severe events and require special attention to limit damage to non-structural elements as well as to avoid problems associated with second-order effects and fracture of connections. Although the damage observed in beam-to-column connections following recent earthquakes has resulted in extensive research on moment-resisting frames,[1] it has also directed further attention back to the benefits of using concentrically braced frames.

According to current seismic design practice, which in Europe is represented by Eurocode 8 (EC8),[2] steel structures may be designed according to either non-dissipative or dissipative behaviour. The former, through which the structure is dimensioned to respond largely in the elastic range, is limited to areas of low seismicity or to structures of special use and importance. Economical design generally necessitates the use of dissipative behaviour through which significant inelastic deformations are accommodated under extreme seismic events. Based on the general format of modern seismic codes, dissipative design is carried out by assigning a structural behaviour factor (also referred to as a force reduction or modification factor), which is used to reduce the code-specified forces resulting from idealised elastic response spectra. This is carried out in conjunction with failure mode control and capacity design procedures, which involve the selection of predefined ductile zones and the provision of overstrength factors for other regions. For concentrically braced frames, capacity design generally implies allowing buckling and yielding in the diagonal bracing members while avoiding this in other frame members and components.

Despite the advantages of using concentrically braced frames, they may not perform satisfactorily in a severe seismic event unless appropriate assessment of the behaviour is carried out and adequate safeguards are provided against the development of undesirable failure mechanisms. During a strong earthquake, bracing members in a concentrically braced frame are subjected to large inelastic deformations in cyclic tension beyond yield and compression into the post-buckling range. This behaviour leads to considerable demands in terms of the selection and detailing of the bracing members as well as the capacity design of other frame components. Moreover, the overall ductility level and distribution offered by various concentrically braced frame configurations need careful examination.

In this paper, the main behavioural aspects and design approaches of typical forms of concentrically braced frames are examined, with emphasis on the conventional types shown in Figs 1(a) and 1(b). Frames of V and inverted-V configurations, such as that in Fig. 1(c), have special features that are beyond the scope of this study. On the other hand, K-braced frames, such as that in Fig. 1(d), are not considered herein as they are not recommended for dissipative design owing to the undesirable damage induced in the columns. It should also be noted that this investigation does not cover recently proposed buckling-resistant brace arrangements,[3] the use of which would entail an evaluation of the trade-off between

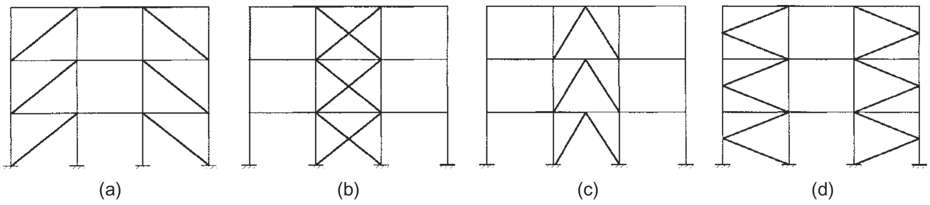

Fig. 1. Typical configurations of concentrically braced frames

performance advantages and additional practical and economical demands.

The design of concentrically braced frames is dealt with in all international codes of practice,[4,5] including EC8.[2] However, there are several inconsistencies between the various codes: some of the differences are associated with the design approaches, whereas others are related to geometric and dimensional limitations. This paper addresses a number of these issues. The primary behavioural aspects of bracing members are discussed first, and the manner through which codes of practice deal with them is described. This is followed by an assessment of the lateral overstrength of frames and its relationship with key response parameters. Selected results from non-linear static and dynamic analyses are also presented, in order to illustrate the influence of bracing member behaviour on the overall capacity and ductility of the structure. Finally, the implications of typical code idealisations for the seismic response are highlighted, and several recommendations to achieve more favourable performance are proposed.

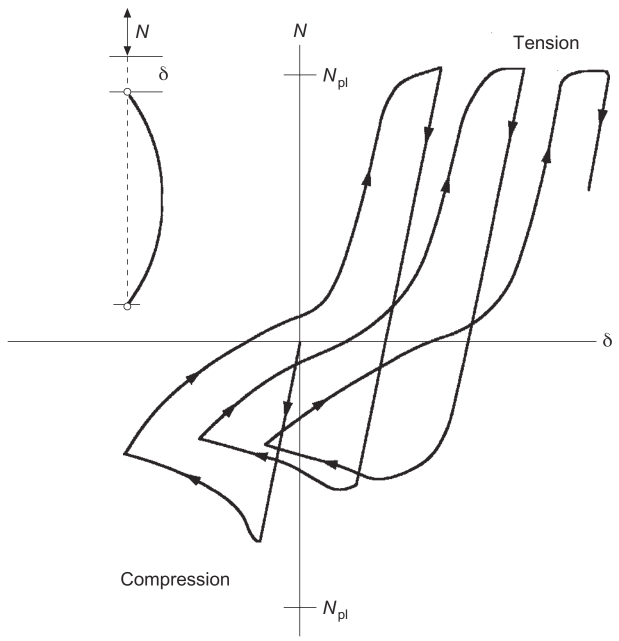

Fig. 2. Axial cyclic response of a typical bracing member

2. MEMBER BEHAVIOUR

2.1. Cyclic response

The response of a concentrically braced frame is typically dominated by the behaviour of its bracing members. Under extreme lateral earthquake loading, the braces experience several cycles of inelastic excursions. This behaviour has been investigated experimentally and analytically by a number of researchers.[6–10]

The hysteretic axial load (*N*) response of a bracing member against axial deformation (δ) is schematically shown in Fig. 2. In compression, member buckling is followed by lateral deflection and the formation of a plastic hinge at mid-length, which leads to a gradual reduction in capacity. On reversing the load, elastic recovery occurs, followed by loading in tension until yielding takes place. Subsequent loading in compression results in buckling at loads lower than the initial strength owing to the residual deflections, the increase in length, and the Bauschinger effect. Moreover, owing to the accumulated permanent elongation, tensile yielding occurs at axial deformations that increase with each cycle of loading. Note, however, that the cyclic behaviour is influenced by several factors, including the member slenderness and the loading history.

Clearly, from a design point of view, it is important to evaluate the tensile and compressive resistance of a bracing member. Additionally, unlike normal design situations in which a conservative lower-bound estimate is sought, appropriate application of capacity design procedures necessitates an evaluation of both lower and upper bounds, as discussed in the following sections.

2.2. Tensile capacity

The nominal tensile resistance is given as the product of the cross-sectional area and the yield strength. However, the actual capacity of the member in tension may be noticeably higher, owing to several factors. The actual yield strength may exceed the design value by up to 20% or more. Furthermore, depending on the ductility demand in tension, strain hardening may result in an increase in the tension force. In a recent comprehensive examination of experimental results on bracing members,[11] this increase was estimated to be between 5% and 10% on average. Another factor is related to the effect of higher strain rate under actual seismic conditions, which may also lead to an additional increase in the yield strength compared with the normal value.

The above-mentioned factors are recognised by most seismic

Equation 6.1 EC8.
Rd>=1.1γov.Rfy. Για γov=1.25
τοτε Rd= 1.37 Rfy.

codes, usually through the provision of an enhancement coefficient ranging between 10% and 35% depending on the code. Realistically, where the evaluation of maximum tensile brace force is necessary in design, partial safety factors for the material should not be applied, and an allowance for the actual yield strength needs to be included.

2.3. Compressive resistance

In compression, overall buckling occurs at a load that depends on the slenderness of the member, which is represented in Eurocode 3 (EC3)[12] by the non-dimensional slenderness, $\bar{\lambda}$. For non-slender cross-sections, $\bar{\lambda}$ is defined as $(N_{pl}/N_{cr})^{0.5}$, in which N_{pl} and N_{cr} are the plastic section capacity and theoretical elastic (Euler) buckling load respectively. The buckling resistance, N_b, is then determined as the product of N_{pl} and χ, which is a slenderness-dependent reduction factor obtained from the appropriate buckling curve, based on the type of section and axis of buckling. Similar column curves are utilised by other international codes of practice, such as CSA[5] and AISC.[13] Fig. 3(a) shows a comparison between the buckling curves proposed in these codes. As indicated in the figure, curve b of EC3 provides a reasonable average, and is therefore selected as a typical representative example of N_b in the assessment carried out herein.

Figure 3(b) depicts the variation of N_b with $\bar{\lambda}$, and also indicates the Euler buckling curve. The value of N_b normally provides a conservative assessment of the minimum buckling strength under monotonic loading, with the possibility of the actual buckling strength being up to 10–15% higher. To account for a possible increase in buckling strength in design checks, codes may apply an enhancement factor, which may be a fixed value of between 10% and 20%.

Under cyclic loading, there is a characteristic reduction in buckling strength after one or two cycles of loading. This has been accounted for in earlier North American provisions[14] by considering a reduced buckling strength N_b' through a relationship of the form

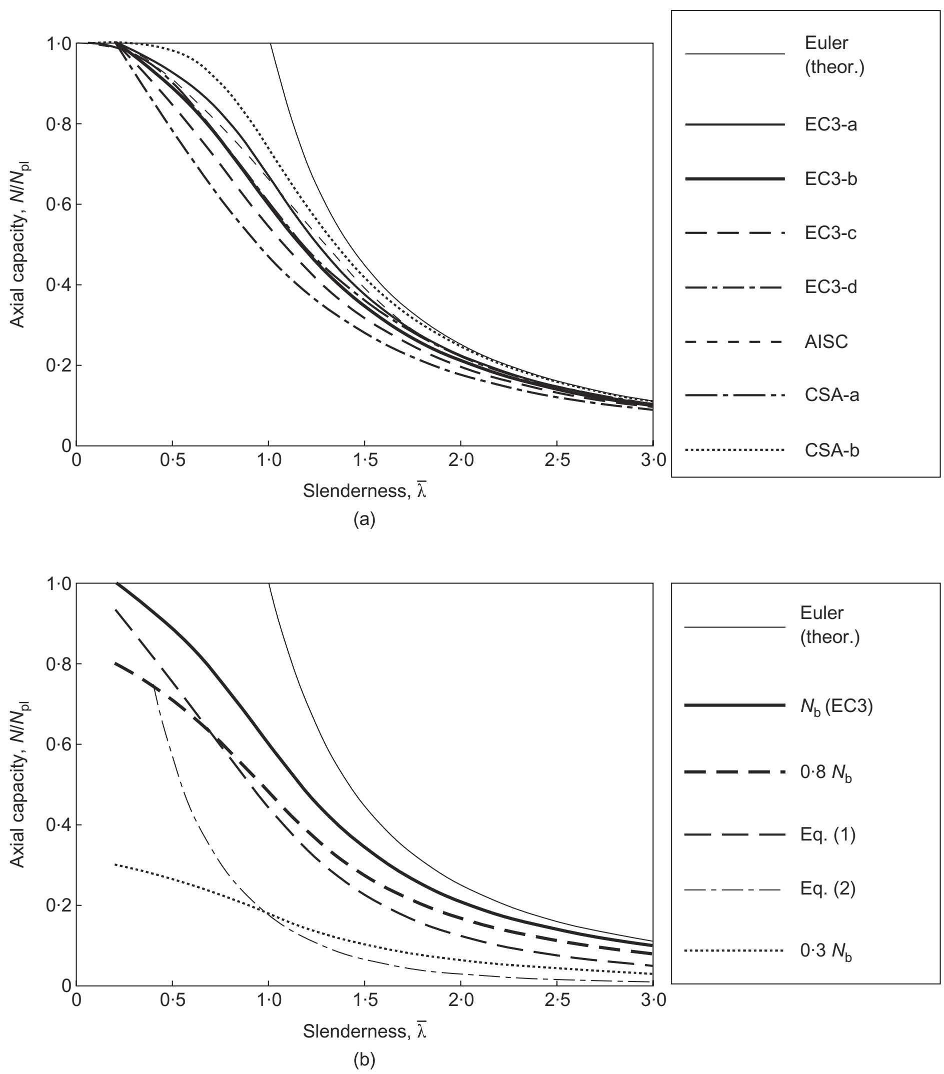

Fig. 3. Compressive resistance of bracing members: (a) comparison of code buckling curves; (b) buckling and post-buckling capacity

reduction of buckling strength after 2 cycles of loading

$$N_b' = \frac{N_b}{1 + 0{\cdot}35\bar{\lambda}} \quad (1)$$

Alternatively, this may be accounted for through a factor of 80%, applied to N_b.[4] As indicated in Fig. 3(b), adoption of either $0{\cdot}8N_b$ or N_b' from equation (1) does not result in substantially different reductions, except for relatively low slenderness values.

It is also important to evaluate the post-buckling resistance of the member, as it has direct implications for the forces developed in other frame members. Typically, the ductility demand can reach up to five times the yield deformation (δ_y). Also, the post-buckling strength does not reduce significantly at higher levels of deformation. A typical relationship[15] for the post-buckling strength at $5\delta_y$, based on a best fit with experimental results, is given as

$$N_{pb} = \frac{0{\cdot}3N_b}{\bar{\lambda}} \quad (\text{for } \bar{\lambda} > 0{\cdot}3) \quad (2)$$

Some codes[4] imply a residual compressive strength of about $0{\cdot}3N_b$ regardless of slenderness, whereas other provisions[16] suggest a factor of 20% of N_{pl}. As shown in Fig. 3(b), the use of either equation (2) or $0{\cdot}3N_b$ does not result in substantially different post-buckling strength for intermediate values of $\bar{\lambda}$, which is likely to be the range of interest for practical design.

2.4. Slenderness limits

In addition to typical limits on the member slenderness (that is, $\bar{\lambda}$ in European practice) for static design situations, seismic codes normally impose upper limits on $\bar{\lambda}$ in order to ensure satisfactory hysteretic behaviour. Under cyclic loading, there is a reduction in the buckling load, and for slender braces the accumulation of plasticity usually leads to low frame resistance and stiffness near the range of zero displacement. This results in severely pinched hysteretic loops and low energy dissipation. For relatively slender braces this behaviour may also cause undesirable shock loading on the structure.

Strict constraints are also usually imposed on the width-to-thickness ratios of the cross-section in order to delay local buckling and brittle fracture. The influence of member slenderness, $\bar{\lambda}$, on fracture has also been subject to investigation. However, whereas earlier studies[10] have indicated that the fracture life of braces generally decreases with an increase in slenderness, more recent work[11] suggests that slender braces can sustain higher ductility levels before fracture occurs. In the latter study, examination of experimental results on braces with hollow rectangular cross-sections indicated that the ductility at fracture is strongly dependent on the slenderness ratio and, to a lesser extent, on the width-to-thickness ratio of the cross-section. This is thought to be a result of the higher compressive strains induced in less slender braces. It is also implied that rectangular hollow sections with $\bar{\lambda}$ lower than unity provide limited ductility levels.

The upper limit on $\bar{\lambda}$ that is recommended by seismic codes generally varies between about 1·3 and 2·0. For example, in EC8,[2] $\bar{\lambda}$ has traditionally been limited to 1·5 in order to prevent early elastic buckling. However, relaxation of this value to 1·8–2·0 is proposed in more recent revisions of the guidelines.[17] In AISC,[4] the member slenderness for ordinary concentrically braced frames (OCBF) is limited to KL/r of $720/\sqrt{F_y}$, where K is the effective length factor, L is the unsupported length, r is the radius of gyration, and F_y is the yield strength (in 1000 lbf/in^2). This slenderness value is equivalent to $\bar{\lambda}$ of about 1·3. For special concentrically braced frames (SCBF), the slenderness limit is increased to $1000/\sqrt{F_y}$, which corresponds to $\bar{\lambda}$ of about 1·8. The relaxation of $\bar{\lambda}$ in SCBF compared with OCBF is in recognition of the more restrictive detailing requirements.

The limits imposed by codes on $\bar{\lambda}$ have a considerable influence on the seismic design of concentrically braced frames. In many cases, it may be the controlling factor in the dimensioning of the bracing members. Most significantly, depending on the design philosophy adopted in the specific code under consideration, $\bar{\lambda}$ of the braces has a direct effect on the performance and design of the overall structure, as discussed in subsequent parts of this study.

3. LATERAL OVERSTRENGTH

3.1. Sources of overstrength

One of the most important characteristics influencing seismic response is the overstrength exhibited by the structure. As indicated in Fig. 4, typical seismic design of regular buildings entails reducing the forces (V_e) obtained from the elastic response spectrum by a behaviour factor (q) in European practice, or force reduction/modification factor (R) in other codes, to arrive at design forces (V_d). The behaviour factor depends on the configuration and expected ductility of the structure under consideration. The actual resistance of the structure (V_y) can, however, be considerably higher than V_d. This reserve strength has significant implications for the ductility demand of critical members as well as the design forces imposed on other structural elements. The presence of overstrength also implies the existence of two different behaviour factors. The first is the one employed in design (that is, V_e/V_d), whereas the second represents the actual force

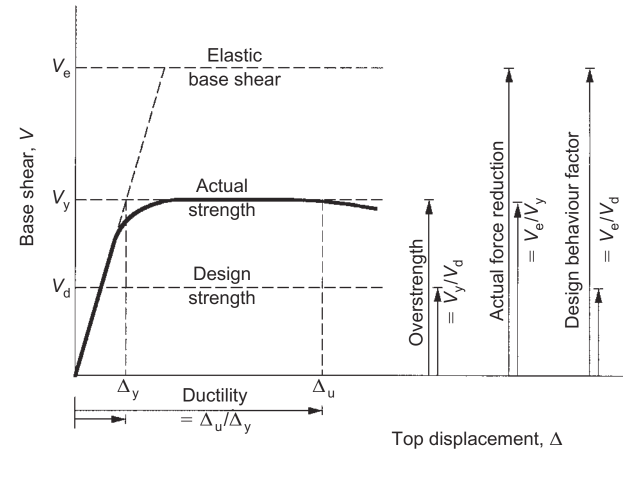

Fig. 4. Relationship between overstrength and behaviour factor

reduction (that is, V_e/V_y), both being directly interrelated through the overstrength (V_y/V_d). Realistically, the maximum overstrength that needs consideration should not exceed the design behaviour factor employed, as entirely elastic behaviour would be implied at this level.

There are several sources that can introduce overstrength in the structure. These include material effects caused by a higher yield stress compared with the characteristic value, or size effects due to the selection of members from standard lists, such as those used for steel sections. Additional factors include the contribution of non-structural elements, or an increase in member sizes due to other load cases or architectural considerations. Most notably, overstrength is often a direct consequence of the simplification of the design approach, particularly in terms of the redistribution of internal forces in the structure.

In the case of concentrically braced frames, the main simplification in the design procedure is related largely to the treatment of buckling and post-buckling in compression. This issue also represents the most noticeable difference in code provisions. Whereas several codes, such as US guidelines,[4] base the design strength on the brace buckling capacity in compression, European practice[2, 17] is contrastingly based on the brace plastic capacity in tension. The significant implications of this inconsistency are addressed in the following sections.

3.2. Compression-based design

As mentioned before, several seismic codes[4,5] base their design strength on the buckling capacity of the braces in compression. To illustrate the implications of this approach, consider the simplified braced frames shown in Fig. 5, which may be regarded as single-storey frames or an idealisation of one storey of a multi-storey structure, ignoring the influence of gravity loads. Using the compression-based philosophy, the design base shear, V_d, corresponds to the attainment of the buckling strength, N_b, in the compression brace, with the tension brace developing a similar value at this stage. Consequently, V_d can be expressed as

$$V_d = 2N_b \cos\theta \tag{3}$$

Beyond this loading stage, the force in the compression brace reduces, whereas that in the tension brace increases until it reaches the tensile plastic capacity, N_{pl}. Clearly, if the design situation requires consideration of upper or lower bounds, N_b may be replaced by N_b' or $1{\cdot}2N_b$ respectively. Similarly, where a maximum value of the tension capacity is sought, N_{pl} may be enhanced by 20%, as discussed in earlier sections.

Assuming that the compressive force is not significantly reduced by the time the tension member yields, coupled with the influence of strain hardening of steel, the ultimate strength realised by the structure can be represented as

$$V_y = (N_{pl} + N_b)\cos\theta \tag{4}$$

Consequently, the overstrength of the frame (V_y/V_d) due to the adoption of this design approach, and without accounting for other overstrength effects, can be determined as

$$\frac{V_y}{V_d} = \frac{N_{pl} + N_b}{2N_b} \tag{5}$$

Clearly, the above overstrength value depends directly on the member slenderness, $\bar{\lambda}$. Using the buckling strength curve of EC3,[12] as discussed previously with respect to Fig. 3(b), the relationship between V_y/V_d and $\bar{\lambda}$ can be directly evaluated, as depicted in Fig. 6. Evidently, for compression-based design, the overstrength increases with slenderness and reaches significant values for very slender members. For satisfaction of capacity design, these higher forces need to be accounted for in the design of frame components and foundations.

From Fig. 6, it is clear that the lateral overstrength due to the compression-based design approach is about 1·5–2·5 for the practical intermediate range of slenderness (that is, corresponding to $\bar{\lambda}$ of about 1·0–2·0, noting that the upper limit imposed by different codes normally varies between 1·3 and 2·0). This overstrength range is comparable to the *system overstrength* factor (Ω_o) of 2·0 (which is used to enhance the lateral seismic actions for the design of members), as recommended in codes such as AISC[4] for concentrically braced frames. However, note that application of Ω_o in the design of

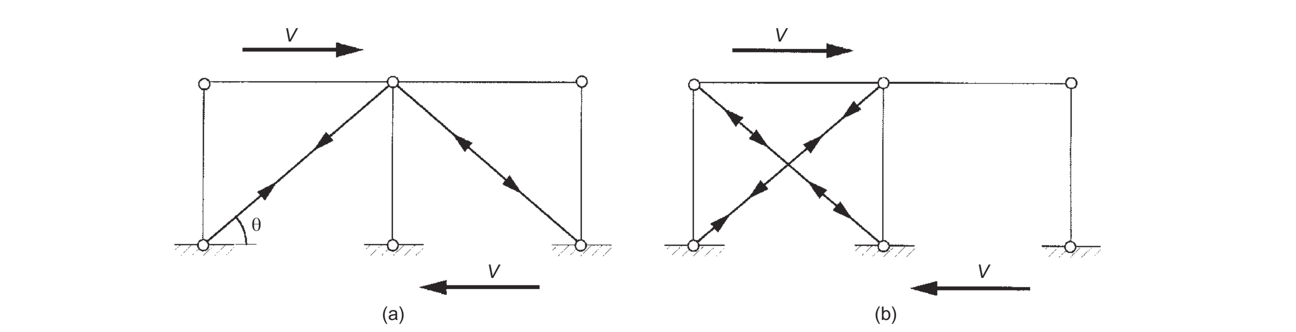

Fig. 5. Idealised single-storey concentrically braced frames: (a) type I; (b) type II

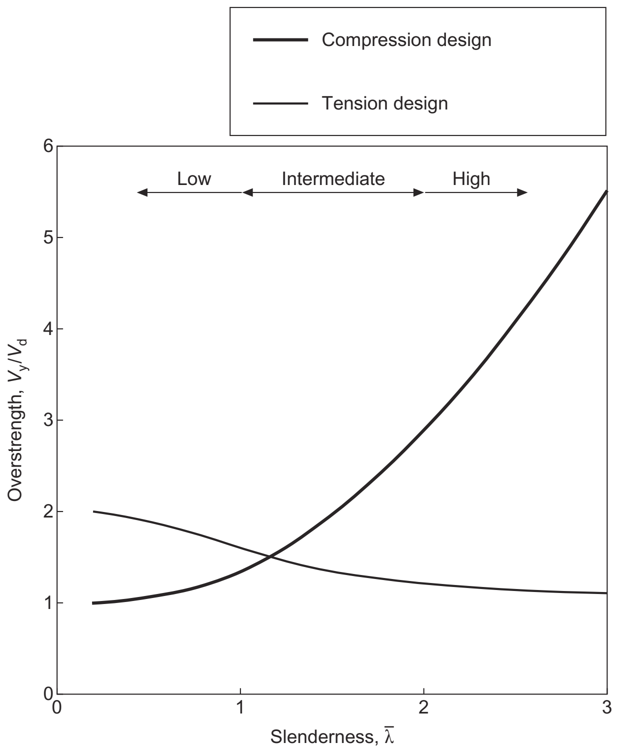

Fig. 6. Relationship between overstrength and slenderness for both design approaches

all frame members, including the braces, may contradict the philosophy of capacity design. This issue is nevertheless not adequately elucidated in these codes.

3.3. Tension-based approach

In European recommendations,[2,17] the lateral loads are based on resistance through the tension braces only. In this respect, this is adopting a tension-only philosophy, which has traditionally been employed in the wind design of relatively slender braces. This approach results in a significant difference in behaviour in comparison with the compression-based idealisation adopted by other codes. Similar to the treatment discussed previously for compression-based design, and considering the frames of Fig. 5, the design base shear for a tension-based approach can be represented as

$$V_d = N_{pl} \cos\theta \tag{6}$$

On the other hand, the ultimate strength of the structure is the same as that estimated in equation (4). Accordingly, the overstrength of the frame (V_y/V_d) based on the tension-approach can be determined as

$$\frac{V_y}{V_d} = \frac{N_{pl} + N_b}{N_{pl}} \tag{7}$$

The relationship between V_y/V_d and $\bar{\lambda}$ for a tension-based design situation is also shown in Fig. 6. Clearly, in this case the overstrength reduces with the increase in slenderness, and becomes relatively insignificant for comparatively large slenderness values.

Unlike other seismic codes,[4,5] EC8[2,17] does not employ system overstrength factors. Based on Fig. 6, this may not be necessary for concentrically braced frames from the viewpoint of the adopted tension-based design approach, except when braces with relatively low slenderness are utilised. On the other hand, in contrast to other codes, EC8 explicitly accounts for the brace overstrength caused by the difference between the brace actual plastic capacity ($N_{pl,Rd}$) and the design axial force in the brace due to seismic actions ($N_{Sd,E}$). The ratio between these two forces ($\Omega = N_{pl,Rd}/N_{Sd,E}$), magnified by 20%, is then used to determine the corresponding seismic actions in other frame members. This *member overstrength* may be relatively large in some design situations if the member size is determined by the limit of $\bar{\lambda}$ rather than by the design force ($N_{Sd,E}$). However, the relaxation of $\bar{\lambda}$ limit in EC8 from 1·5 in earlier guidelines[2] to 2·0 in more recent provisions[17] has led to a significant mitigation of this effect.

3.4. Forces in other components

As mentioned previously, capacity design of concentrically braced frames entails designing the structural elements, other than the braces, to respond primarily in the elastic range without experiencing yielding or buckling. To achieve this, design forces should be determined with due account taken of overstrength. These maximum forces in the frame elements would, however, depend on the member location as well as on the frame configuration.

As an example of the above, the reaction of the internal column of the frame shown in Fig. 5(a) needs to be obtained based on the maximum possible force in the tension brace in conjunction with the minimum post-buckling load in the compression brace, i.e. $(N_{pl} - N_{pb})\sin\theta$. In contrast, the same reaction in the frame of Fig. 5(b) necessitates consideration of the maximum force in the tensile brace together with the maximum buckling strength of the compression brace—that is, $(N_{pl} + N_b)\sin\theta$. This is illustrated in Fig. 7, where the reaction of the internal column, normalised to $N_{pl}\sin\theta$, is plotted against $\bar{\lambda}$ for the two simple frames of Fig. 5, in which N_{pb} is considered as $0{\cdot}3N_b$. Clearly, this reaction need not exceed the value consistent with the elastic forces (that is, with $q = 1$).

Note also, with reference to Fig. 7, that ignoring the force in the compression brace in determining the reaction of the internal column is conservative for the frame in Fig. 5(a), but would lead to unsafe design of lower floor columns and foundations for the frame in Fig. 5(b). As a means of reducing this effect, recent revisions of EC8[17] suggest imposing a lower limit of 1·3 on $\bar{\lambda}$ when X-braced configurations are considered.

Similar critical loading conditions need to be accounted for in various design situations such that simplified procedures are based on conservative assumptions. It would probably be impractical for codes of practice to place application rules to cover all possible combinations. Consequently, the designer should be aware of the underlying assumptions and simplifications in order to enable a valid implementation of the principles of capacity design and failure mode control.

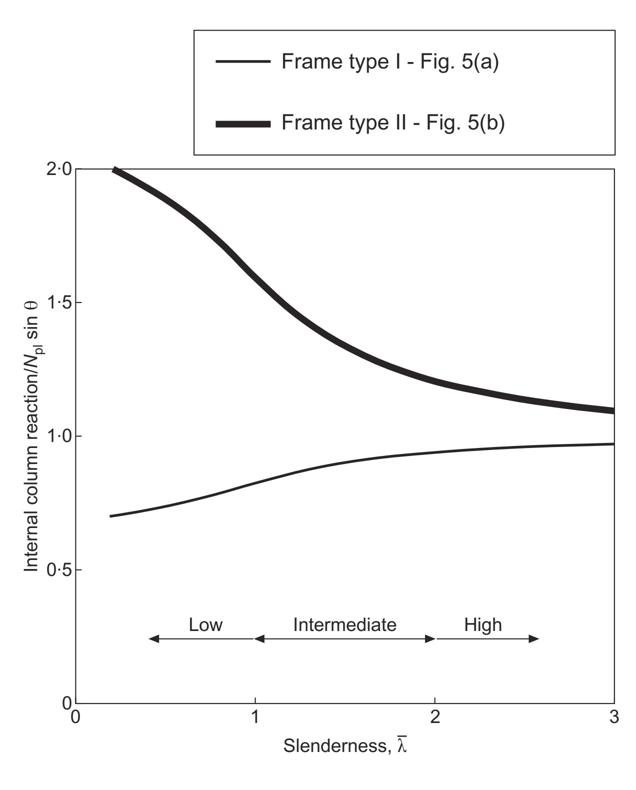

Fig. 7. Normalised internal column reaction for the two idealised frames

3.5. Ductility demand

As described in the previous section, the lateral overstrength largely determines the maximum forces that develop in various frame members. In addition, the overstrength has significant effects on the dynamic response of a frame. First, the stiffness of the frame is directly related to the actual size of the bracing members. In turn, the natural period is closely associated with the square root of the stiffness. Most importantly, the overstrength has a considerable influence on the ductility demand of the frame.

The interrelationship between overstrength, stiffness, period and ductility demand may be estimated using approximate expressions based on simple elasto-plastic systems.[18] Considering that, in practice, the fundamental natural period of a concentrically braced frame is likely to be in the period range of 0·1–0·6 s, the ductility demand (μ) can be estimated, using an equal energy approach, from

$$\mu = \frac{(V_e/V_y)^2 + 1}{2} \tag{8}$$

For longer-period structures, the ductility demand is more linearly related to V_e/V_y. Within a period range of 0·1–0·6 s, the idealised design response spectrum would typically have a constant amplification. Accordingly, from a design standpoint, variation of period within this range would not lead to a modification of the elastic forces (V_e) or the design base shear (V_d). Consequently, assuming the validity of equation (8), and with reference to Figs 4 and 6, an approximate estimation of the ductility demand may be deduced for the two design approaches and for various values of the design behaviour factor (q), as depicted in Fig. 8.

The curves shown in Fig. 8 are believed to provide a reasonable prediction of the ductility demand of an idealised concentrically braced frame of the form shown in Fig. 5. Clearly, the ductility demand is significantly influenced by the existing overstrength, which depends on $\bar{\lambda}$ and the design approach adopted. As shown in Fig. 8, the presence of relatively high overstrength leads to considerable reductions in the ductility demand.

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It is useful to compare the results of Fig. 8 with the design provisions in seismic codes. In EC8,[2,17] a behaviour factor of 4·0 is recommended for concentrically braced frames. Moreover, by assuming the same factor for predicting inelastic drifts, the code is implying a ductility level of 4·0. With reference to Fig. 8, it appears that, for the tension-design used in Eurocode 8, and for q of 4·0, the ductility demand exceeds the code prediction for $\bar{\lambda}$ larger than 1·2. On the other hand, AISC[4] suggests a behaviour factor of 6·0 and 5·0 for special and ordinary concentrically braced frames respectively. The code also specifies displacement enhancement factors, which imply ductility demands of 5·0 and 4·5 for the two frame classes respectively. For the compression design employed by the codes, Fig. 8 suggests that this is satisfactory for $\bar{\lambda}$ larger than 1·5, but could significantly underestimate the demand for lower values of slenderness.

The above assessments illustrate the influence of the two different design approaches to brace buckling on the level of lateral overstrength of the frame and, in turn, on the actual forces attained in various frame members as well as on the expected ductility demand. Further treatment of these issues and their implication on the overall frame performance is

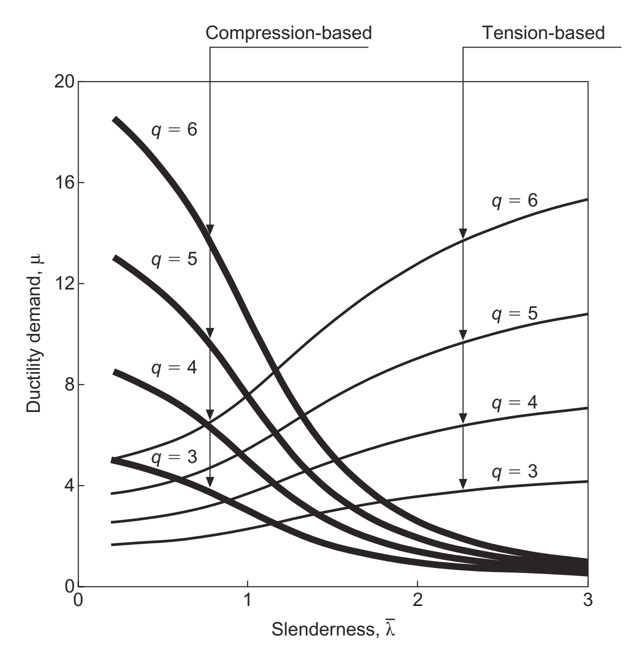

Fig. 8. Predicted ductility demand based on the two design approaches

discussed within the analytical examination presented in the following section.

4. FRAME PERFORMANCE

4.1. Analytical modelling

In order to illustrate the main behavioural aspects discussed above, and to allow further examination of salient response parameters, selected non-linear static and dynamic analyses are carried out on idealised concentrically braced frames. The analysis is performed using the non-linear finite element program ADAPTIC,[19] which has been extensively verified in previous studies.[20,21] The program accounts for geometric and material non-linearities, and includes an extensive library of elements and material models.

Throughout this analytical investigation the elasto-plastic element, which employs a cubic shape function, is utilised. This element formulation is based on a distributed plasticity approach that models the spread of plasticity within the cross-section and along the length. The element response is assembled from contributions at two Gauss points, where the cross-section is discretised into a number of monitoring areas. The formulation utilises a relationship between the direct material stresses and strains, and allows various material models to be included.

For the frames considered in this study, the brace members are represented by eight cubic elements to achieve a high level of accuracy. A kinematic bilinear material model for steel is employed, with a strain hardening ratio of 0·5%, elastic modulus of 210×10^3 N/mm^2 and yield stress of 300 N/mm^2. All the results are, however, presented in a normalised format to reduce their dependence on the specific geometric and material parameters of the selected frames.

In modelling the brace members, geometric imperfections may be introduced as an explicit initial out-of-straightness or through the concept of equivalent notional loads, both of which provide similar results.[22] In this study, the latter approach is used to represent a geometric imperfection of 1/1000 of the brace length. Moreover, for modelling purposes, the braces are assumed to have rectangular cross-sections in order to facilitate control on variations of the member slenderness and capacity.

4.2. Static behaviour

The behaviour of a single-storey frame, of the form shown in Fig. 5(a), is first examined in order to emphasise the behavioural aspects discussed in previous sections. The response of the frame in Fig. 5(b) would have similar trends, but requires more careful consideration in determining the in- and out-of-plane buckling length.[23] The modelled frame is assumed to have two spans of 5·0 m each and a height of 3·5 m. As the lateral response of the structure is determined mainly by the diagonal bracings, the beams and columns may realistically be modelled using rigid links, or alternatively by cubic beam elements provided it is ensured that buckling or yielding do not occur in these members. Note also that the results are not noticeably affected by the extent of the gravity loads applied.

The single-storey frame is first subjected to a monotonically increasing lateral displacement at the top. The analysis is repeated with braces exhibiting $\bar{\lambda}$ values between zero and 3·0. Fig. 9 depicts the axial load (normalised to N_{pl}) in the braces with the increase in storey drift (normalised to the storey height). The results are presented up to drifts of 3·0%, as upper limits for the ultimate state normally range between 2·0% and 3·0%. As expected, the tension brace (or stocky compressive brace) typically yields at a drift of 0·3–0·5%. With the increase in slenderness, the buckling load (N_b) decreases considerably, and exhibits similar trends to the design buckling curve presented in Fig. 3(b). In practice, the division between elastic and inelastic buckling corresponds to a slenderness of about 1·3–1·5, rather than the theoretical unity, depending on the level of imperfections and residual stresses.

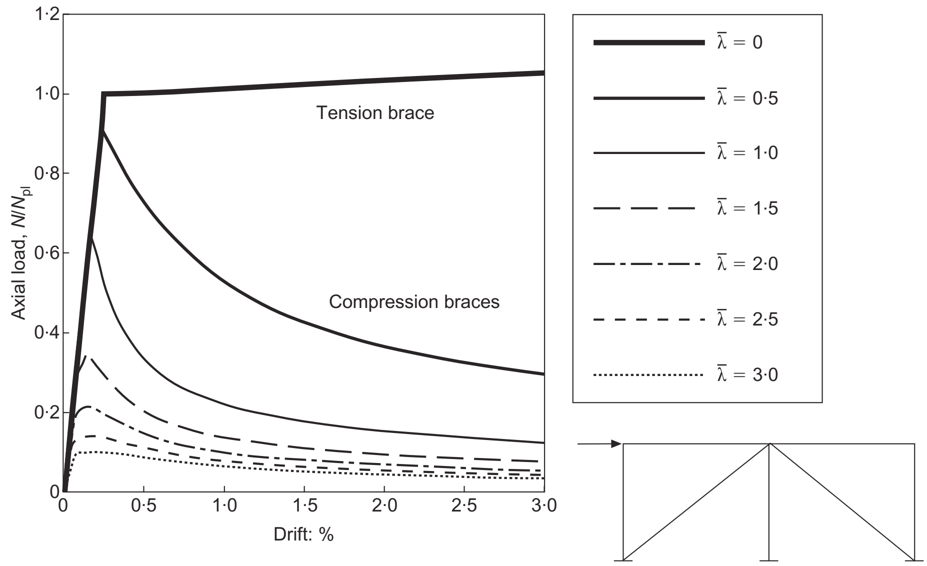

Fig. 9. Monotonic axial response of bracing members

The same frame is subjected to a static cyclic displacement of increasing amplitude, and three brace slenderness values of 0·5, 1·5 and 2·5 are considered. Fig. 10(a) depicts the normalised axial load in the brace, which is loaded first in compression, plotted against the frame drift. On the other hand, the base shear of the frame (normalised by $N_{pl} \cos \theta$) is shown in Fig. 10(b). As discussed before, it is evident from Fig. 10 that stocky braces have more favourable hysteretic behaviour than relatively slender members.

4.3. Dynamic response

As discussed previously, the approach employed for design has a direct influence on the ductility demand of the frame. To examine this analytically, the idealised single-storey frame used in the static analysis is subjected to 6 s of the lateral ground acceleration from the El Centro earthquake at Imperial Valley. A mass of 20 000 kg and a viscous damping ratio of 5% are assumed. The compression- and tension-based approaches are both separately employed in dimensioning the braces for variations of $\bar{\lambda}$.

For the selected earthquake, the analytical results obtained seemed to be in general agreement with the trends of ductility demand indicated in Fig. 8, within an expected discrepancy range of about 20%. For example, Figs 11(a) and 11(b) depict the lateral displacement history, normalised to that at yield, of two frames dimensioned according to both design approaches, for $\bar{\lambda}$ of 0·5 and 2·0 respectively, assuming a q factor of 4·0. Evidently, for relatively large slenderness values, the ductility demand associated with tension design is considerably larger than that for compression design. The opposite trend occurs for low slenderness, although such low values may not be of wide application in building frames.

The ductility demands predicted in Fig. 8 should, however, be treated as a general guide rather than as a precise assessment. Owing to the dependence of the observed ductility demand on the input excitation and the dynamic characteristics of the structure, a more accurate evaluation would require a more extensive investigation involving various frame configurations and a wide range of earthquake records.

For design purposes, the predicted ductility demand needs to be met by that available in the braces. The ultimate ductility provided by a bracing member is, however, dependent on the type of cross-section as well as on $\bar{\lambda}$. For example, some experimental studies[11] indicate that the fracture life of braces of rectangular hollow cross-sections increases with higher values of $\bar{\lambda}$. In addition, the ductility demand imposed on a concentrically braced frame needs to be restricted to values consistent with the code-defined inter-storey drift limits. Given that brace yielding typically occurs at drifts between 0·3% and 0·5%, a ductility of about $5\delta_y$ would imply drifts within the range of the 1·5–3·0% limits commonly employed by seismic codes. Moreover, the relatively high ductility demands should be avoided in order to limit the extent of in- and out-of-plane lateral deformation of the braces. Unless specifically accounted for in design, these movements can cause considerable damage to non-structural elements such as walls and cladding.

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4.4. Inelastic distribution

The distribution of inelasticity is an issue that merits particular attention in concentrically braced frames. First, owing to the asymmetry between the tensile capacity and buckling resistance of the braces, inelastic drifts may occur disproportionately in one lateral direction, depending on the characteristics of the excitation. Accordingly, seismic codes[2,4,5] normally include specific rules in order to limit the discrepancy between the brace capacity in both lateral directions across the width and breadth of a building.[24] This helps in providing a closer balance in lateral resistance, which assists in reducing

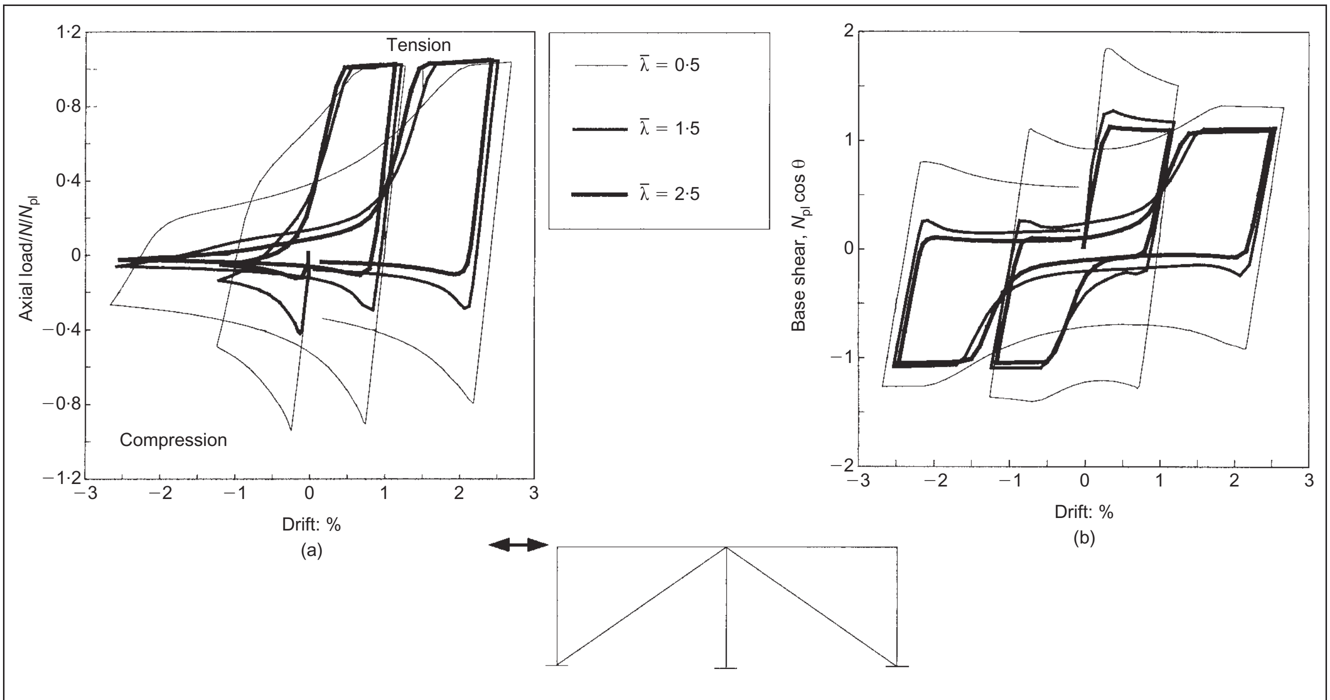

Fig. 10. Cyclic response of bracing member and overall frame: (a) brace behaviour; (b) frame response

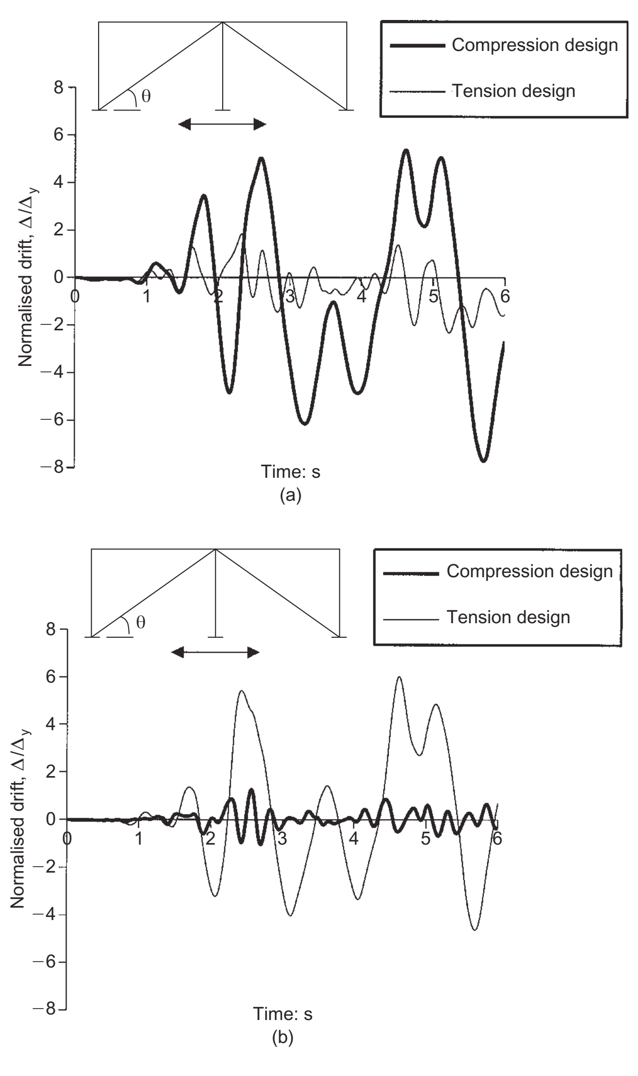

Fig. 11. Response using both design approaches for stocky and slender braces: (a) $\bar{\lambda} = 0{\cdot}5$; (b) $\bar{\lambda} = 2{\cdot}0$

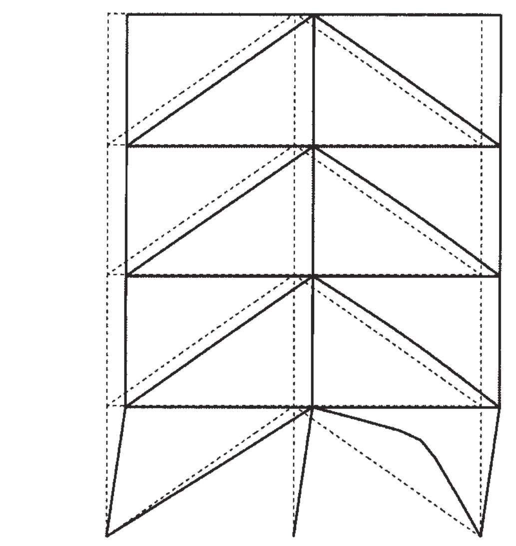

Fig. 12. Deformed shape of frame with soft first storey

the accumulation of inelastic drifts that could occur in one direction more than the other.

More significantly, the low post-yield tangent stiffness of the braces can lead to a concentration of inelasticity in one level over the height of a multi-storey frame. This behaviour would take place even when buckling was delayed or prevented. It could also occur even when brace capacities were relatively well balanced with demands over the height, as required in recent revisions of EC8,[17] which suggests limiting the maximum difference in brace overstrength (Ω) to 25%. The likelihood of concentration of demand has also prompted more recent North American code revisions[16,25] to propose limits on the height or number of stories in concentrically braced frames.

Under static *push-over* loading simulating first-mode response, a mechanism will occur at a storey as soon as the braces start yielding at that level. If the brace area is constant over the height, the mechanism will occur at the first storey, as shown in Fig. 12. Similar behaviour is also observed under realistic earthquake loads, although a possible contribution from higher modes may have an influence on the response.

In order to illustrate the significance of this aspect of behaviour, the idealised frame utilised in the previous section is extended to four storeys and subjected to the El Centro excitation at the base. Four specific design situations are considered:

(*a*) braces with constant area over height
(*b*) braces with variable area over height to match the capacity demand
(*c*) constant brace area and continuous column
(*d*) variable brace area and continuous column.

In cases (*a*) and (*b*), the frames are modelled using the commonly used approach in which all the nodes are assumed to be pinned. On the other hand, in (*c*) and (*d*), the columns are assumed to be continuous over the height, and are analytically represented by four cubic elements in each storey. The column size is selected to satisfy capacity design requirements, with due account taken of typical gravity loads. Note also that, where a continuous column is utilised, the moments induced in the columns need to be considered in design.

In terms of the braces, frames (*a*) and (*c*) have constant area, whereas in (*b*) and (*d*) the brace area is reduced in upper storeys to match the demand imposed by an idealised first mode response (that is, an inverted triangular distribution). In all braces, $\bar{\lambda}$ is retained at 1·5, the mass at each floor is specified as 20 000 kg, and the damping ratio is considered to be 5% of the critical.

The inter-storey drifts normalised to the storey height, at each level of the four frames, are depicted in Fig. 13, and Fig. 14 shows the total energy dissipated in the braces at each level, normalised to $N_{pl}\,\delta_y$ of the first storey brace. Moreover, the envelopes of inter-storey drift and energy dissipation are extracted and presented in Fig. 15. By examining Figs 13–15, it is clear that adopting a variable brace capacity does lead to an improvement in inelastic distribution. However, this improvement is only marginal if it

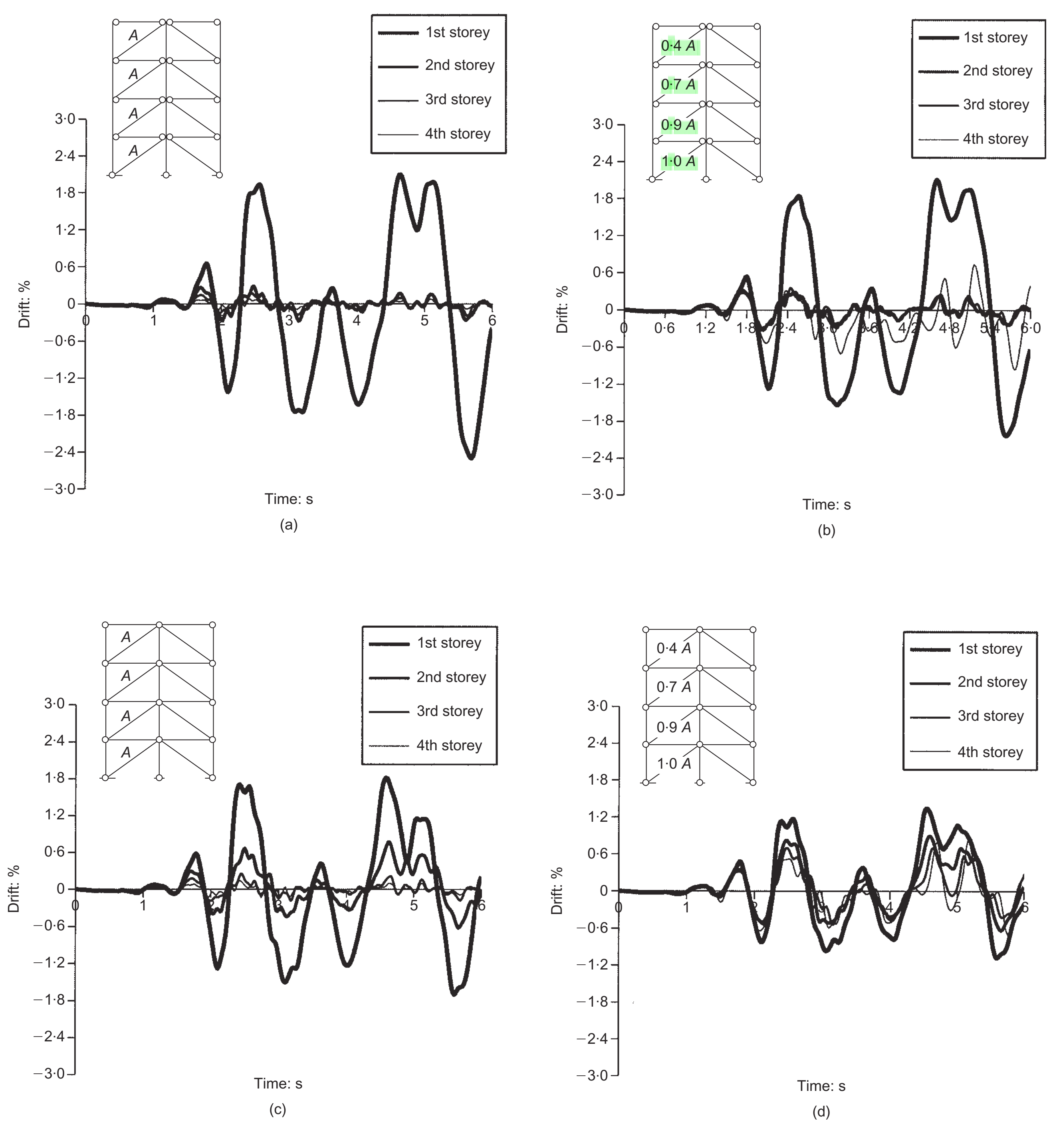

Fig. 13. Distribution of inter-storey drift over height: (a) constant braces; (b) variable braces; (c) constant braces (with continuous columns); (d) variable braces (with continuous columns)

is not combined with a continuous column. Similarly, utilising a continuous column with a constant brace over the height is not as effective as when combined with variable braces.

Although the distribution of ductility demand would also depend on the dynamic characteristics of the structure and the excitation, it is evident that a considerably more favourable distribution of inelastic demand can generally be achieved by employing a balanced brace capacity over the height in conjunction with continuous columns. These measures need to be explicitly incorporated within capacity design procedures in order to safeguard against the adverse implications of excessive and localised ductility demands.

5. CONCLUSIONS

This paper assesses the seismic design procedures for concentrically braced frames. The main behavioural issues associated with capacity design procedures are discussed, and

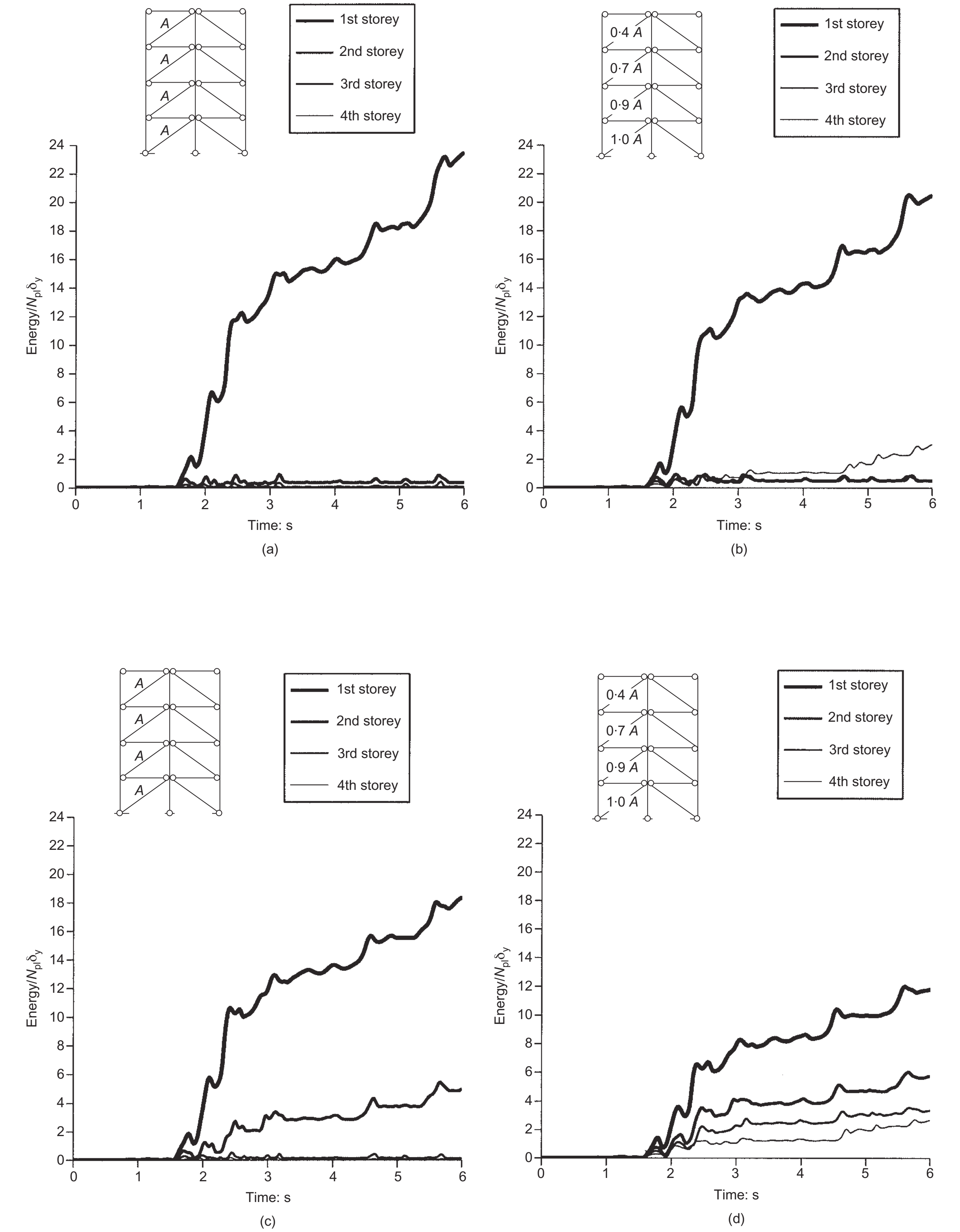

Fig. 14. Distribution of energy dissipation over height: (a) constant braces; (b) variable braces; (c) constant braces (with continuous columns); (d) variable braces (with continuous columns)

related European and North American provisions are critically examined. In particular, the study focuses on the implications of the adopted design idealisation on the lateral frame overstrength, which is shown to have a direct influence on important design aspects and response characteristics. Owing to the need to employ simplified practical approaches in design guidelines, some of the issues addressed in this study are often inadequately treated or overlooked. Apart from the continuous

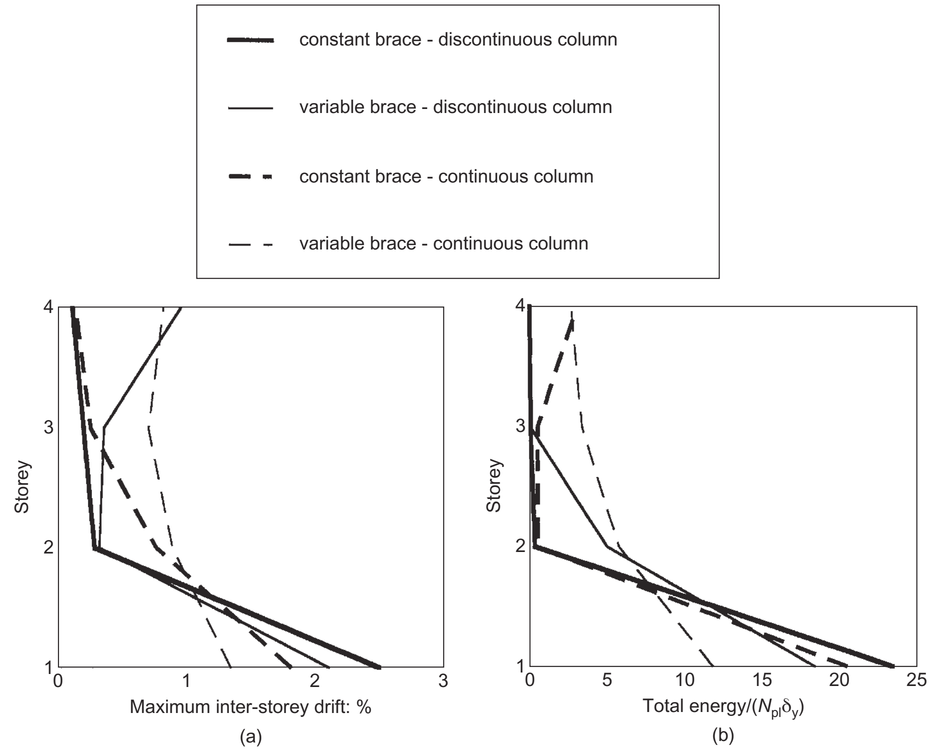

Fig. 15. Distribution of inter-storey drift and total energy envelopes over height: (a) inter-storey drift; (b) total energy

requirement for code improvement, development and harmonisation, it is also imperative for the designer to have an insight into the key behavioural considerations.

For appropriate implementation of capacity design principles in concentrically braced frames, both the maximum and minimum axial capacities of the brace members need to be assessed. This involves the tensile capacity, the buckling strength and the post-buckling resistance at the expected level of ductility. The lower bound should be considered in evaluating the *dependable* axial strength, whereas the upper bound would be required for determining the axial *overstrength*, depending on the design situation. The appropriateness of using available expressions is assessed within the discussions, particularly with respect to the brace slenderness. For design purposes, an enhancement or reduction factor of 20% appears to be appropriate for estimating maximum or minimum values of compressive strength respectively. On the other hand, the post-buckling strength can be evaluated as 30% of the buckling load. These upper and lower bound axial capacities represent actions that need to be checked in the design of other frame components such as connections, beams, columns and foundations.

The important role played by structural overstrength is emphasised in this study. For concentrically braced frames, the main source of overstrength is related to the design simplification in considering buckling of bracing members. The inconsistency between the tension-based approach adopted in European guidelines and the compression-based design used in other codes results in considerable differences in the main response characteristics. For intermediate and large values of slenderness, the overstrength produced from compression-based design can be significantly higher than that from the tension-based approach. Whereas tension design has potential merits in terms of economy and lower forces on other frame components, compression design has several advantages in terms of reduced ductility demand and lower post-buckling deformation. The situation is reversed for braces of relatively low slenderness, which may not be of wide application in building structures. Where the tension design approach is used, very low behaviour factors should be employed, preferably 3 or lower, in order to limit the extent of ductility demand. For both approaches, careful consideration of the frame overstrength needs to be undertaken in determining the forces imposed on other components. In this respect, the *member overstrength* procedure adopted in European practice allows a more explicit implementation of capacity design concepts, in the case of concentrically braced frames, compared with the *system overstrength* practice used in other codes.

The analytical studies performed illustrate the influence of the brace slenderness on the static and dynamic response of typical frame configurations. In particular, the characteristic unfavourable hysteretic behaviour of frames with relatively slender braces is discussed. The low frame resistance and stiffness near the range of zero displacement results in severely pinched loops and poor energy dissipation as well as the potential for inducing undesirable shock loading on the structure. Consequently, codes appropriately impose an upper limit on slenderness, which, with due consideration of practical and economic aspects, is converging to a value of about

1·8–2·0 in current seismic codes. This limit is particularly important for tension-based design, for which the ductility demand increases with higher slenderness.

The asymmetry of ductility demand in the lateral direction is addressed in codes of practice, but the vulnerability of concentrically braced frames to localisation of inelasticity over the height requires further attention. It is shown through illustrative dynamic analysis that additional measures need to be explicitly introduced in design provisions. This is best achieved by utilising a balanced capacity-to-demand brace design over the height in conjunction with an appropriate continuous column detail. In general, unfavourable performance may occur as a result of these possible concentrations coupled with the undesirable effects of significant post-buckling brace deformations such as local buckling, susceptibility to fracture and damage to non-structural components. Accordingly, design entailing relatively high ductility demands should normally be avoided in concentrically braced frames unless adequate inelastic dynamic behaviour is ensured and demonstrated.

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